

Retrofit of Seismically Deficient RC Columns with Textile-Reinforced Mortar (TRM) Jackets*

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Summary: The effectiveness of a new structural material, namely textile-reinforced mortar (TRM), was investigated experimentally in this study as a means of confining old-type reinforced concrete columns with limited capacity due to bar buckling or due to bond failure at lap splice regions. Comparisons with equal stiffness and strength fiber-reinforced polymer (FRP) jackets allow for the evaluation of the effectiveness of TRM versus FRP. Tests were carried out on full scale non-seismically detailed RC columns subjected to cyclic uniaxial flexure under constant axial load. Thirteen cantilever-type specimens with either continuous longitudinal reinforcement (smooth or deformed) or lap splicing of longitudinal bars at the floor level were constructed and tested. Experimental results indicated that TRM jacketing is quite effective as a means of increasing the cyclic deformation capacity of old-type RC columns with poor detailing, by delaying bar buckling and by preventing splitting bond failures in columns with lap spliced bars. Compared with their FRP counterparts, TRM jackets used in this study were found to be equally effective in terms of increasing both the strength and deformation capacity of the retrofitted columns. From the response of specimens tested in this study, it can be concluded that TRM jacketing is an extremely promising solution for the confinement of reinforced concrete columns, including poorly detailed ones with or without lap splices in seismic regions.

1 Introduction and Background

The upgrading of existing reinforced concrete (RC) structures through jacketing of columns has become a very popular technique in an increasingly large number of rehabilitation projects, both seismic and non-seismic. The use of fiber-reinforced polymers (FRP) has gained considerable popularity among all jacketing techniques, due to the favorable properties offered by these materials, namely high strength to weight ratio, corrosion resistance, ease and

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speed of application, and minimal change of geometry. Despite all these advantages, the FRP retrofitting technique has a few drawbacks, e.g. poor behavior at high temperatures; high costs; inapplicability on wet surfaces or at low temperatures; hazards for the manual worker, even though modern epoxies gradually become less hazardous due to smaller solvent contents; lack of vapour permeability, which may cause damage to the concrete structure; and difficulty to conduct post-earthquake assessment behind FRP jackets. These are mainly attributed to the organic epoxy resins used to bind the fibers. An interesting alternative to FRP materials are the so-called Textile-Reinforced Mortars (TRM) TRIANTAFILLOU ET AL. [1]. These new materials are made of textiles, that is fabric meshes made of long woven, knitted or even unwoven fiber rovings in at least two directions, impregnated with inorganic binders, such as cement-based mortars. The density, that is the quantity and the spacing, of rovings in each direction can be controlled independently, thus affecting the mechanical characteristics of the textile and the degree of penetration of the mortar matrix through the mesh.

Studies on the use of textiles as reinforcing materials of cement-based products commenced in the early 1980s, but developments in this field progressed slowly until the late 1990s. However, during the past few years the research community has put substantial effort on the use of textiles as reinforcement of cementitious products, mainly in new constructions (e.g. CURBACH AND JESSE [2], BRAMESHUBER ET AL. [3], HEGGER AND VOSS [4]). Research on the use of textiles in the upgrading of concrete structures has been limited. Most of the studies concern the bond between concrete and cement-based textile composites, as well as flexural or shear strengthening of beams (CURBACH AND ORTLEPP [5], BRUECKNER ET AL. [6], TRIANTAFILLOU AND PAPANICOLAOU [7]). In these studies it was concluded that properly designed textiles combined with cement-based binders have a good potential as strengthening materials of RC members. The first study reported in the literature on the use of textiles in combination with cementitious binders for the confinement of concrete is that of TRIANTAFILLOU ET AL. [1]. In this study the authors investigated experimentally the application of TRM as a means of increasing the axial strength of plain concrete through confinement and they compared the behavior of TRM-confined cylinders and prisms with that of specimens confined with FRP jackets of equal stiffness and strength. Main conclusions were that: (a) TRM jacketing provides a substantial increase in compressive strength and deformation capacity of plain concrete; and (b) compared with their FRP counterparts, TRM jackets may result in slightly reduced effectiveness.

In a more recent study, BOURNAS ET AL. [8] investigated experimentally the use of TRM jackets as a means of confining poorly detailed RC columns, which suffer from limited deformation capacity due to buckling of the longitudinal bars. Tests were carried out both on short prisms under concentric compression, reproducing the behavior of compression zones in RC members where bar buckling is critical, and on nearly full scale non-seismically detailed RC columns with smooth bars subjected to cyclic uniaxial flexure under constant axial load. All specimens retrofitted with TRM jackets had their FRP-retrofitted counterpart, which enabled comparisons of the two systems. Main conclusion in this study was that TRM jacketing is quite effective (and equally to its FRP counterpart) as a means of increasing the

cyclic deformation capacity and the energy dissipation of old-type RC columns with poor detailing, by delaying bar buckling.

In the present study the authors go one step further by investigating experimentally the use of TRM jackets as a means of confining poorly detailed old-type RC columns with smooth or deformed rebars, which suffer from limited deformation capacity under seismic loads due to either buckling of the longitudinal bars or bond failure at lap splice regions.

2 Experimental Program

2.1 Test specimens and experimental parameters

A total of 13 full-scale reinforced concrete column specimens with the same geometry comprising 7 columns with continuous longitudinal reinforcement (smooth or deformed) and 6 columns with lap splicing of longitudinal bars at the floor level were constructed and tested under cyclic lateral load (Fig. 1a). The specimens were flexure-dominated cantilevers with a height to the point of application of the load (shear span) of 1.6 m (half a typical story height) and a cross section of 250x250 mm. The columns were fixed into a heavily reinforced 0.5 m-deep base block, 1.2x0.5 m in plan, within which the longitudinal bars were anchored with 90-degree hooks at the bottom. To represent old-type non-seismically designed and detailed columns, both series of continuous and spliced specimens were reinforced longitudinally with four 14 mm-diameter deformed bars and 8 mm diameter smooth stirrups at a spacing of 200 mm, closed with 90-degree hooks at both ends with an overlap length of 100 mm. The geometry of a typical cross section is shown in Fig. 1b. Note that the specimens with smooth bars which were comprised the preliminary part of this research had different concrete cover (25 mm) in comparison with the specimens reinforced with deformed bars (10 mm), which were constructed at a later stage. Naturally, this dissimilarity does not affect the comparison of test results, as the last is performed independently for smooth and deformed longitudinal reinforcement.

The specimens which were constructed with continuous longitudinal reinforcement comprised 3 columns with smooth bars (Series L0S_...) and 4 specimens with deformed bars (Series L0_...). Two specimens were tested without retrofitting, as controls (L0S_C and L0_C), the second pair was retrofitted with a double-layered CFRP jacket (L0S_R2 and L0_R2), the third pair of specimens was retrofitted with an equal (to its FRP counterpart) stiffness and strength carbon fiber TRM jacket comprising 4 layers (L0S_M4 and L0_M4), and the last specimen with deformed bars was retrofitted with a lower stiffness and strength four-layered glass fiber TRM jacket (L0_M4G), which represents a rehabilitation solution of lower cost in comparison with specimen L0_R2 and L0_M4 in practical strengthening applications.

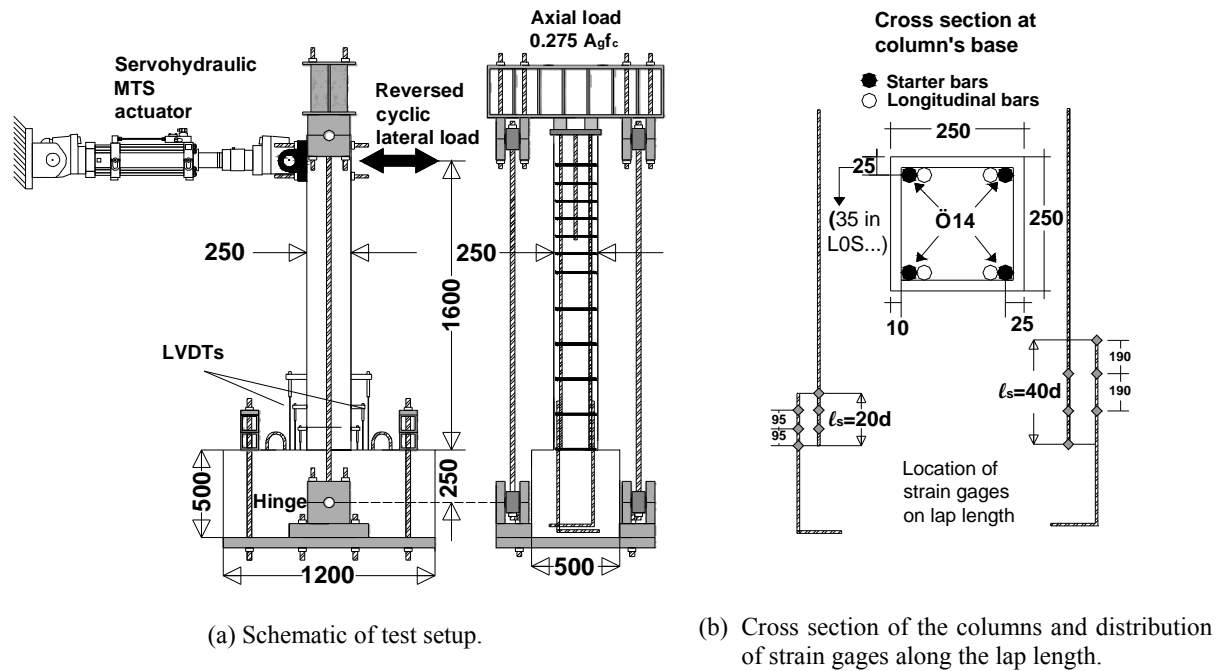


Fig. 1: Test set up and specimens geometry (dimensions in mm).

The effectiveness of TRM versus FRP jackets, applied at the ends of old-type RC columns for specimens constructed with lap splicing of longitudinal reinforcement above the column base, was evaluated for two different lap lengths, which were selected equal to 20 and 40 bar diameters, as shown in Fig. 1b. Columns with the shorter lap lengths (Series L20d_...) are more representative of RC construction up to the late 1970s. These columns were designed as follows: One specimen was tested without retrofitting as control (L20d_C), the second one was retrofitted with a two-layered CFRP jacket (specimen L20d_R2) and the third one was retrofitted with an equal (to its FRP counterpart) stiffness and strength carbon fiber TRM jacket comprising four layers (specimen L20d_M4). Columns with longer lap lengths (Series L40d_...) are more representative of RC construction up to the late 1990s. These columns were given the notation L40d_C, L40d_R2 and L40d_M4, that is identical to Series L20d_..., except for the lap length.

Note that the layers in the TRM-jacketed columns were twice as many compared with their FRP counterparts, resulting in two “equivalent” confining systems, that is with equal stiffness and strength in the circumferential direction (as explained below, the fibers of the two jacketing systems in the circumferential direction were of the same type and nearly twice as many in the FRP system compared with the TRM system).

In summary, the notation of specimens is LX_YN, where X defines the lap splice length above the column base (0 for continuous reinforcement, 20d for a lap splice length of 20 rebar diameters, 40d for a lap splice length of 40 rebar diameters), Y denotes the type of jacket

(C for the unjacketed - control columns, R for resin-based jackets and M for mortar-based jackets) and N denotes the number of layers. For specimens with smooth longitudinal bars the letter S (smooth) was added after letter X, while for the specimen strengthened with a glass fiber TRM jacket the letter G was added after letter N. The jackets extended from the base of each column (a gap of about 10 mm was left) to a height of 430 mm except for the two columns with longer lap splices (L40d_R2 and L40d_M4) where the jackets were extended to a height of 600 mm. The overlapping length of the jacket was equal to 150 mm. Prior to jacketing, the four corners of the columns which received jacketing were rounded at a radius equal to 25 mm. A summary of the experimental parameters and retrofitting schemes is presented in Table 1.

Table 1: Experimental Parameters

Specimen notation	Type of longitudinal reinforcement	Lap Length	Concrete Strength f_c [MPa]	Confinement with composite materials		
				Jacketing materials	Fibers	No. of layers/Jacket's height
L0S_C	Smooth	--	29.8	--	--	--
L0S_R2	Smooth	--	30.3	FRP	Carbon	2 / 430 mm
L0S_M4	Smooth	--	30.5	TRM	Carbon	4 / 430 mm
L0_C	Deformed	--	28.9	--	--	--
L0_R2	Deformed	--	28.6	FRP	Carbon	2 / 430 mm
L0_M4	Deformed	--	28.4	TRM	Carbon	4 / 430 mm
L0_M4G	Deformed	--	28.3	TRM	Glass	4 / 430 mm
L20d_C	Deformed	20d _b = 280 mm	27.8	--	--	--
L20d_R2	Deformed	20d _b = 280 mm	26.5	FRP	Carbon	2 / 430 mm
L20d_M4	Deformed	20d _b = 280 mm	26.3	TRM	Carbon	4 / 430 mm
L40d_C	Deformed	40d _b = 560 mm	25.8	--	--	--
L40d_R2	Deformed	40d _b = 560 mm	25.5	FRP	Carbon	2 / 600 mm
L40d_M4	Deformed	40d _b = 560 mm	25.3	TRM	Carbon	4 / 600 mm

2.2 Materials and Strengthening Procedures

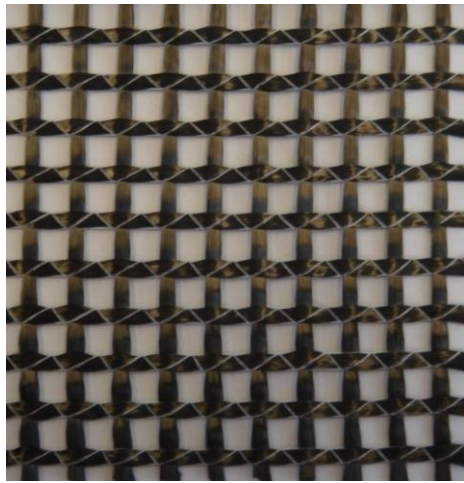
The longitudinal smooth bars had a yield stress of 372 MPa, a tensile strength of 433 MPa and an ultimate strain equal to 17%, while the corresponding values for deformed bars were 523 MPa, 624 MPa and 12% (average values from six specimens). The corresponding values for the steel used for stirrups were 351 MPa, 444 MPa and 19.5%. In order to simulate field conditions the base blocks and the columns were cast with separate batches of ready-mix concrete (on two consecutive days). Casting of the columns was made with separate batches too, due to the unavailability of a large number of moulds. The compressive strengths on the day of testing the columns, measured on 150x150 mm cubes (average values from three specimens), are presented in Table 1 for all columns. Cylinders with a diameter of 150 mm and a height of 300 mm were also used to obtain the splitting tensile strength of the concrete; the

average tensile strength which was obtained from six specimens on the day of testing the columns was equal to 3 MPa.

For the specimens receiving TRM jacketing (L0S_M4, L0_M4, L0_M4_G, L20d_M4, L40d_M4) two commercial textiles with equal quantity of carbon or glass rovings in two orthogonal directions were used (Fig. 2a). Each fiber roving was 3 mm wide and the clear spacing between rovings was 7 mm. The weight of carbon and glass fibers in the textiles was 348 g/m^2 and 480 g/m^2 , respectively, corresponding to a nominal thickness for each layer (based on the equivalent smeared distribution of fibers) equal to 0.095 mm and 0.089 mm, respectively. The mean tensile strength of the carbon and glass fibers (as well as of the textiles, when the nominal thickness is used) was taken from data sheets equal to 3800 MPa and 1700 MPa, respectively. The elastic modulus of carbon and glass fibers was 225 GPa and 70 GPa, respectively. For the specimens receiving FRP jacketing (L0S_R2, L0_R2, L20d_R2, L40d_R2), a commercial unidirectional carbon fiber sheet was used, with a weight of 300 g/m^2 and a nominal thickness of 0.17 mm. For the specimens receiving mortar as a binding material, a commercial inorganic dry binder was used, consisting of cement and polymers at a ratio of about 8:1 by weight. The water:binder ratio in the mortar was 0.23:1 by weight, resulting in plastic consistency and good workability. Finally, for the specimens receiving resin adhesive bonding, a commercial structural adhesive (two-part epoxy resin with a mixing ratio 3:1 by weight) was used with a tensile strength of 70 MPa and an elastic modulus of 3.2 GPa (cured for 7 days at 23 °C). The adhesive had low viscosity such that complete wetting of the sheets was possible by using a plastic roller.

Application of the mortar was made in approximately 2 mm thick layers with a smooth metal trowel. After application of the first mortar layer on the (dampened) concrete surface, the textile was applied and pressed slightly into the mortar, which protruded through all the perforations between fiber rovings. The next mortar layer covered the textile completely and the operation was repeated until all textile layers were applied and covered by the mortar. Of crucial importance in this method, as in the case of epoxy resins, was the application of each mortar layer while the previous one was still in a fresh state. A photograph of the application method of textile combined with mortar binder to provide jacketing in one of the specimens used in this study is shown in Fig. 2b.

The strength of mortar used in this study was obtained through flexural and compression testing according to EN 1015-11, using a servohydraulic MTS testing machine. Flexural testing was carried out on three 40x40x160 mm hardened mortar prisms, at an age of 28 days. The prisms were prepared and cured in the laboratory until testing, in conditions identical to those for the jackets used for confinement (except for the first two days, when the prisms were inside the moulds). The prisms were subjected to three-point bending at a span of 100 mm and from the peak load the flexural strength was calculated. Compression testing was carried out on each of the fractured parts using two 40x40 mm bearing steel platens on top and bottom of each specimen. The average flexural and compressive strength values were 6.51 MPa and 20.8 MPa, respectively.



(a) Photograph of textile used in this study.



(b) Application of the TRM jacket.

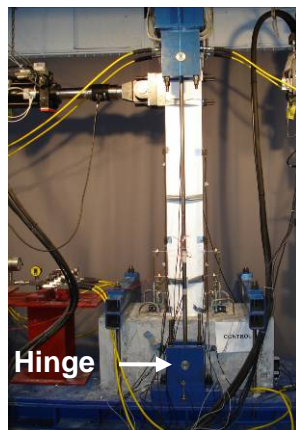
Fig. 2: Textile used and strengthening procedure.

2.3 Experimental Setup and Procedure

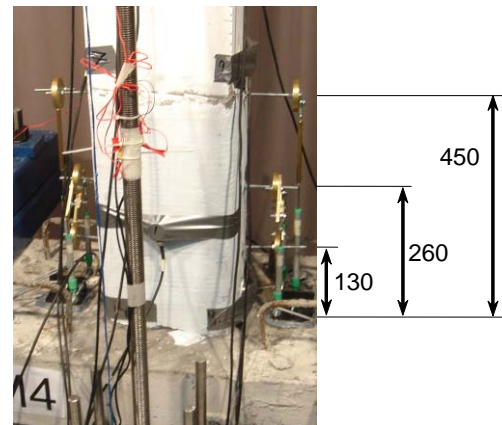
To simulate seismic excitation, the columns were subjected to lateral cyclic loading which consisted of successive cycles progressively increasing by 5 mm of displacement amplitudes in each direction. The loading rate was in the range from 0.2 mm/sec to 1.1 mm/sec, the higher rate corresponding to higher displacement amplitude, all in displacement-control mode. At the same time a constant axial compressive load was applied to the columns, corresponding to 27.5% of the members' compressive strength (depending on concrete strength, this load ranged from 435 kN to 523 kN). The lateral load was applied using a horizontally positioned 250 kN MTS actuator and the axial load was exerted by a set of four hydraulic cylinders with automated pressure self-adjustment, acting against two vertical rods connected to the strong floor of the testing frame through a hinge (Fig. 1a and 3a). With this set-up the $P-\Delta$ moment at the base section of the column is equal to the axial load times the top displacement (that is at piston fixing position) of the column, times the ratio of hinge distance from the base (0.25 m) and the top (0.25+1.60=1.85 m) of the column (that is times $0.25/1.85 = 0.135$).

Displacements, rotations and curvatures at the plastic hinge region were monitored using six rectilinear displacement transducers (three on each side, perpendicular to the piston axis) fixed at cross sections 1, 2 and 3, with a distance equal to $\ell_1=130$ mm, $\ell_2=260$ mm and

$\ell_3=450$ mm, respectively, from the column base, as shown in Fig. 3b. The instrumentation also comprised a total of 8 strain gages for each column with continuous longitudinal reinforcement and a total of 12 strain gages for each column with lap splices. For the columns with continuous bars two strain gages were mounted on each reinforcing bar (at a height of 100 mm from the base cross section) in order to estimate the strain of longitudinal bars at the onset of buckling. This location was chosen as buckling is expected to occur at the mid-height between the first two stirrups above the column base. The twelve strain gages of the columns with lap splices were mounted on one pair of lapped bars (starter-longitudinal) per column side as follows (Fig. 1b): (a) Three along the starter bars, at distances from the column base equal to 0 mm, 95 mm and 190 mm (for Series L20d_...), or 0 mm, 195 mm and 390 mm (for Series L40d_...). (b) Three along the longitudinal bars, at distances from the column base equal to 95 mm, 190 mm and 280mm (for Series L20d_...), or 195 mm, 390 mm and 560mm (for Series L40d_...). Measurements from the strain gages on each pair of starter-longitudinal bars were used to determine the strain distribution of bars and bond stresses along the splice length.



(a) Photograph of test set up.



(b) Position of displacement transducers.

Fig. 3: Instrumentation of the test set up (dimensions in mm).

3 Test Results and Discussion

3.1 General Remarks

The response of all columns tested is given in Fig. 4 in the form of load-drift ratio (obtained by dividing the top displacement with the column's height) loops. Key results are also presented in Table 2, which includes: (a) The peak resistance in the two directions of loading. (b) The drift ratio corresponding to peak resistance in the two directions of loading. (c) The drift ratio at conventional "failure" of the column, defined as reduction of peak resistance in a

cycle below 80% of the maximum recorded resistance in that direction of loading. For some specimens (L0S_R2, L0S_M4, L0_M4, L40d_R2, L40d_M4) the reduction of peak resistance when the stroke of the horizontal positioned actuator was exhausted (at a drift ratio of 7.81%) was less than 20% of the maximum recorded resistance in both directions of loading. In such cases the drift ratio at conventional “failure” is simply stated as >7.81%. (e) The observed failure mode.

Table 2: Summary of Test Results

Specimen notation	Peak force (kN)		Drift at peak force (%)		Drift at “failure” (%)		Failure Mode
	Push	Pull	Push	Pull	Push	Pull	
L0S_C	37.9	-39.8	2.8	2.8	3.7	3.7	Buckling of longitudinal bars
L0S_R2	40.2	-41.7	3.4	4.4	>7.8	>7.8	Conventional failure was not reached
L0S_M4	37.2	-41.6	4.1	4.4	>7.8	>7.8	Conventional failure was not reached
L0_C	41.6	-42.5	2.5	2.5	3.4	3.4	Buckling of longitudinal bars
L0_R2	43.7	-48.7	2.8	3.1	5.0	5.3	Buckling of longitudinal bars above FRP jacket
L0_M4	45.8	-49.2	2.8	2.8	>7.8	>7.8	Conventional failure was not reached
L0_M4G	48.8	-45.3	4.0	2.8	7.5	6.9	Fracture of the TRM jacket due to both rebars buckling and concrete dilation
L20d_C	41.5	-36.6	1.9	1.9	4.1	3.1	Splitting bond failure followed by spalling of the concrete cover
L20d_R2	41.3	-52.9	2.8	3.1	5.3	6.2	Splitting longitudinal cracking followed by pull out bond failure of lapped bars
L20d_M4	48.5	-49.8	3.1	2.2	5.0	5.0	Splitting longitudinal cracking followed by pull out bond failure of lapped bars
L40d_C	46.3	-43.8	2.5	2.2	3.4	3.1	Splitting bond failure followed by spalling of the concrete cover
L40d_R2	43.0	-49.9	4.7	5.0	>7.8	>7.8	Conventional failure was not reached
L40d_M4	45.9	-50.5	1.9	3.7	>7.8	>7.8	Conventional failure was not reached

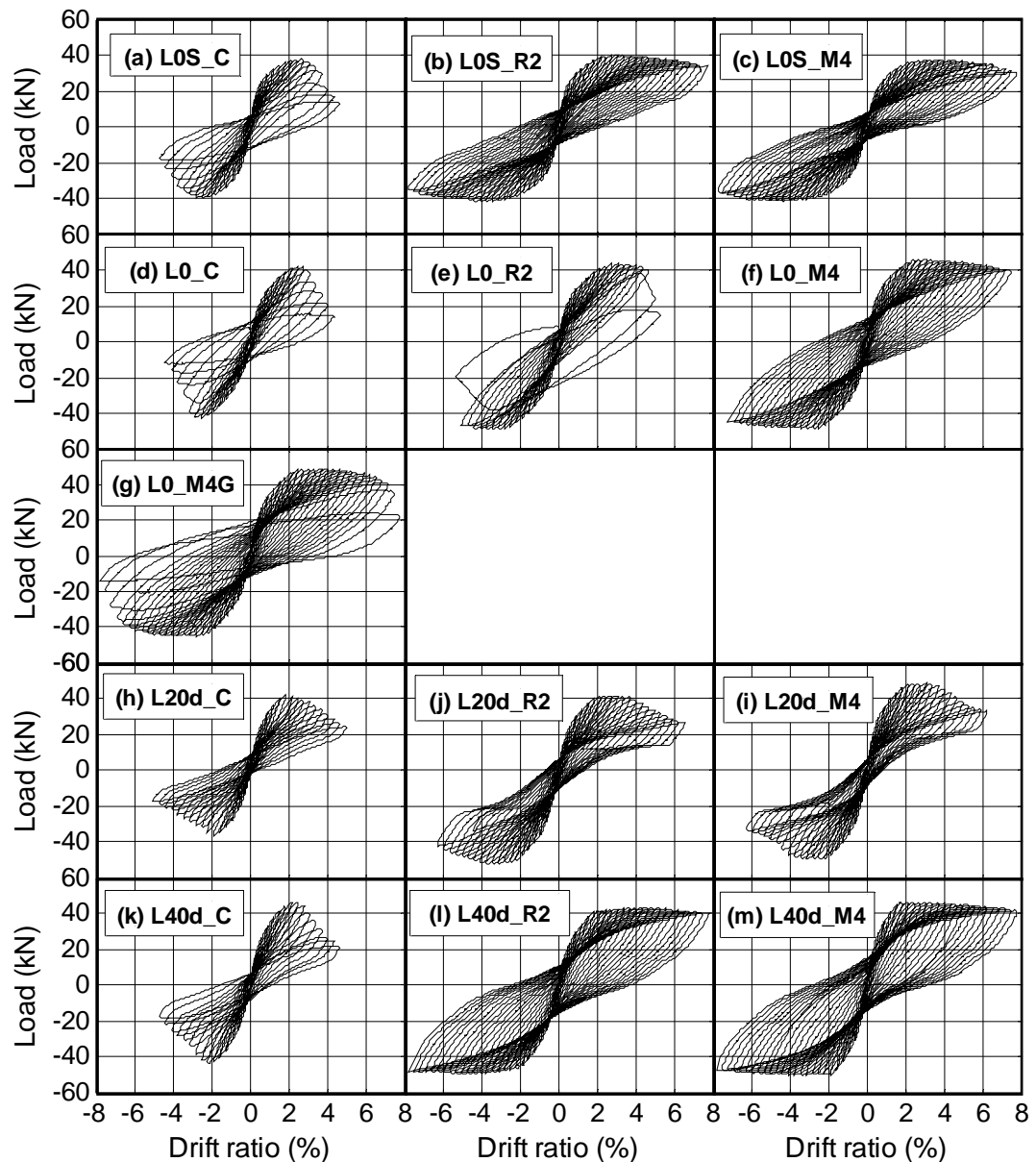


Fig. 4: Load versus drift ratio curves for all specimens tested

3.2 Specimens with Continuous Longitudinal Reinforcement

The performance and failure mode of all tested specimens with continuous longitudinal reinforcement was controlled by flexure. Buckling of longitudinal bars initiated thereupon their yielding (next loading cycle) for each specimen. The failure mode of the two unretrofitted specimens L0S_C and L0_C was controlled by buckling of longitudinal rebars above the column base, which led to strength degradation. The outward bending of buckled bars at the column's corners (at mid-height between the two adjacent stirrups closest to the column

base) was found to be responsible for the concrete cover spalling over the lower 200 mm of the column (Fig. 5a). The drift ratio at failure sustained by the two unretrofitted columns was 3.75% and 3.43%, respectively.

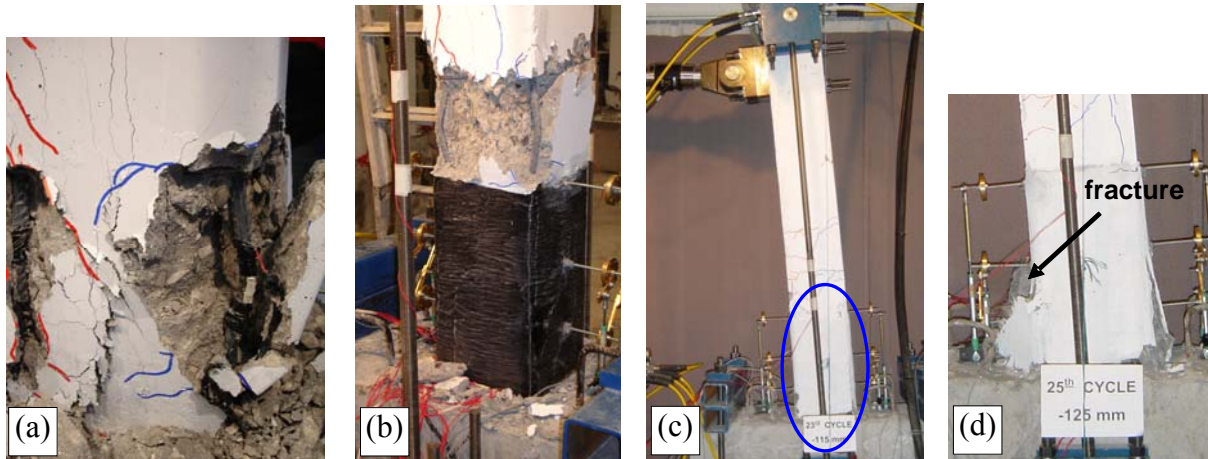


Fig. 5: (a) Disintegration of concrete and bar buckling. (b) Buckling of longitudinal bars above the FRP jacket. (c) Undamaged carbon TRM jacket at end of test. (d) Fracture of glass TRM jacket due to bar buckling.

The behavior of the two retrofitted columns with smooth bars (L0S_R2, L0S_M4) was very similar (Fig. 4b and 4c), but quite different from and far better than their unretrofitted counterpart L0S_C (Fig. 4a). Member deformation capacity increased by a factor of more than two, while peak resistance was practically the same as in the unretrofitted column; furthermore the post peak response was quite stable, displaying very gradual strength degradation. Whereas the FRP jacket in column L0S_R2 exhibited limited rupture over the lower 50 mm at 7.2% drift ratio, the TRM jacket remained intact until the test was terminated at 7.8% drift ratio.

The behavior and failure mode of retrofitted columns with deformed longitudinal bars (L0_R2, L0_M4, L0_M4G) was not controlled by bar buckling above the column base. According to measurements of strain gages placed on longitudinal bars inside the jackets, it was observed that bar buckling was not averted for FRP or TRM confined specimens; it developed with a significant delay, ranging from 3-7 cycles with respect to their unretrofitted counterpart (L0_C), without lateral strength degradation. This is attributed to the behavior of buckled bars under external confinement. These bars could sustain a significant part of their compressive load after buckling as the concrete cover spalling remained in place and provided lateral support.

The confinement provided by the FRP jacket to specimen L0_R2 restrained the outward bending of longitudinal bars inside the FRP jacket region. Owing to this fact the concrete cover dilation was marginal and a large amount of strain energy was stored in the confined concrete without any stress relaxation in the compression zone. This resulted in the transition

of the compressive force above the FRP jacket, where buckling of longitudinal bars finally occurred abruptly in the space between the FRP jacket's end and the next stirrup at a height of 530 mm (Fig. 5b). Similar observations of bar buckling above the FRP jacket (in regions with significant lower bending moment than that of the column base) have been made by other researchers too (e.g. BOUSIAS ET AL. [9]).

Contrary to specimen L0_R2, rebar buckling in columns L0_M4 and L0_M4G developed gradually inside the TRM-jacketed area, as the compressive force released from early buckled bars was carried by the surrounded confined concrete inside the jackets. This is possible to occur in this confining system, as TRM jackets are able to deform outwards without early fiber rupture, due to the low composite action between fibers and mortar, which allows for higher local deformations (e.g. slip of fibers within rovings).

In specimen L0_M4 the carbon fiber TRM jacket remained intact until the test was terminated at drift ratio equal to 7.81% (Fig. 5c), while in specimen L0_M4G fracture of the glass fiber TRM jacket (at a drift ratio equal to 7.2%) led to failure (Fig. 5d). Fracture of the jacket initiated from a limited number of fiber bundles when the hoop stresses reached their tensile capacity, and then propagated rather slowly in the neighbouring bundles, as a result of concrete dilation and outward bending of the longitudinal rebars a little below the jacket's mid-height; as a result of this gradual fiber fracture, the respective failure mode was ductile.

Overall, the behavior of carbon and glass TRM jacketed specimens was very similar, but quite different from and far better than that of the FRP confined and unretrofitted specimens. Member deformation capacity increased by a factor of 1.5, 2.3 and 2.1 for specimens L0_R2, L0_M4 and L0_M4G, respectively, in comparison with the control specimen, corresponding to drift ratios at failure equal to 5.15%, 7.81% and 7.2%; this indicates a higher effectiveness of TRM versus FRP jackets, by about 50%. Peak resistance was practically the same for all jacketed specimens and about 10% higher than that of the control specimen, which experienced bar buckling at earlier stages of deformation.

3.3 Specimens with Lap-Spliced Longitudinal Reinforcement

The performance and failure mode of all specimens with lap splices was also controlled by flexure. Significant longitudinal and horizontal splitting cracks were developed along the splice length of lapped bars for both unretrofitted specimens L20d_C (Fig 6a) and L40d_C (Fig 6b) at drift ratios of 1.56% and 2.5%, respectively, corresponding to peak lateral load. The length and width of the longitudinal cracks along the splice length was increasing at higher drift levels as the bond between reinforcing bars and concrete was deteriorating. As a consequence of this, the concrete under compression spalled (Fig. 6c) along the lower (approximately) 100 mm and 175 mm from the base of specimens L20d_C and L40d_C respectively, leading to substantial lateral strength degradation after peak lateral load. Contrary to control specimen L0_C with continuous bars, the expansive spalling of the concrete in the

critical zone was not followed by buckling of longitudinal rebars for two reasons: first, the compression reinforcement was doubled; second, the quick strength degradation of the specimens associated with the extensive bond deterioration reduced the demand of the compression reinforcement to resist the applied load. The drift ratio at failure (average values for both loading directions) sustained by unretrofitted columns L20d_C and L40d_C was 3.59% and 3.28%, respectively. The corresponding values for the bond strength (τ) between spliced bars and the surrounded concrete were 4.4 MPa and 2.5 MPa. The latter was established according to the discrete strain readings along the splice length (detailed calculation of the bond stress distribution along the lap splice are given in BOURNAS [10]).

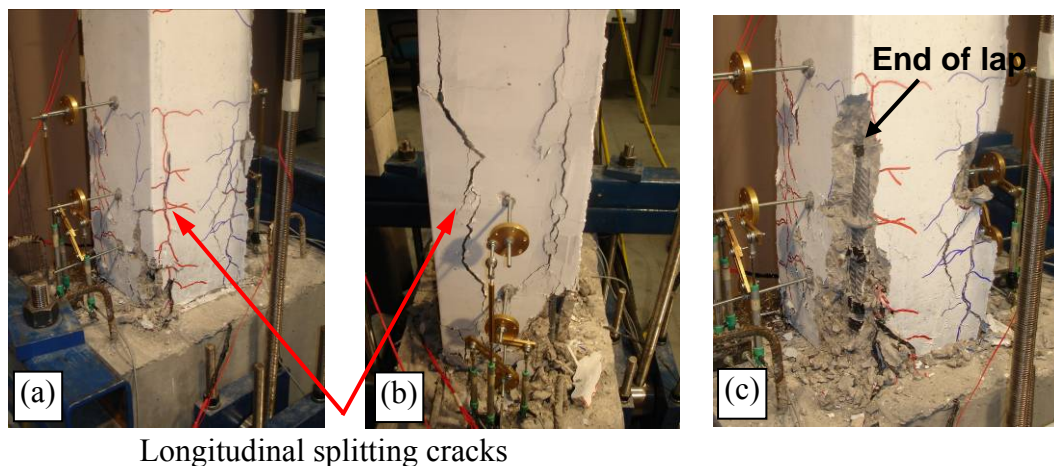


Fig. 6: Longitudinal splitting cracks for specimens (a) L20d_C and (b) L40d_C. (c) Failure of unretrofitted column L20d_C.

TRM and FRP jacketed columns, with either short or long lap length, responded far better than their unretrofitted counterparts both in terms of strength and deformation capacity at failure. Confinement provided to the columns sufficient resistance against splitting cracks and lateral expansion of concrete. Thus spalling of the concrete cover was controlled and the slip along the splice length was progressively increased in proportion to the horizontal displacement without significant bond stress deterioration.

The effectiveness of confinement was limited in the case of short lap length equal to 20 bar diameters in comparison with that of the long lap length equal to 40 bar diameters for both TRM and FRP confining systems. Specimens L20d_R2 and L20d_M4 (with short lap lengths) sustained reversed deformation cycles up to 6.3% drift before failing due to pull-out bond failure of the spliced bars at an average bond strength (in both directions of loading) between lap-spliced bars and concrete of 6.8 MPa and 6.4 MPa, respectively. Pull-out bond failure occurred when longitudinal splitting cracks had propagated along the entire splice length; thus at that point the presence of TRM or FRP jacket had no effect on the residual splice capacity. Contrary to specimens with short lap splices, in specimens L40d_R2 and

L40d_M4 where the calculated bond stresses were much lower, namely 3.1 MPa and 2.9 MPa, respectively, bond failures and spalling of concrete were suppressed until the end of the test at a drift ratio of 7.81%.

For columns L20d_R2 and L20d_M4 the mean strength increase (in both directions of loading) for both confining systems was 20.3% and 25.6%, respectively, in comparison with the control specimen (L20d_C), while the corresponding increase in deformation capacity was 64.7% and 38.8%, respectively. Columns with longer lap splices (L40d_R2 and L40d_M4) behaved in an identical manner until the end of the test at a drift ratio of 7.81% (maximum stroke of piston was reached), resulting in an increase of the members' deformation capacity by a factor of more than 2.5. Peak resistance was practically the same as in the unretrofitted column, indicating that a lap splice length of 40 diameters is adequate for the development of the columns' full strength. Overall, it may be concluded that TRM confining jackets provide substantial gain in lateral strength and deformation capacity of cyclically loaded RC rectangular columns with lap splices at the columns' base. Compared with equal stiffness and strength FRP jackets, they are characterized by a slightly reduced effectiveness in terms of deformation capacity for columns with short lap splices and with the same effectiveness for columns with longer lap lengths.

4 Comparison of Test Results with Code Formulations

The cyclic deformation capacity of RC columns, a key property in displacement-based design used in seismic rehabilitation applications, is typically expressed through the members' attained drift ratio at failure. This important parameter for all specimens tested is compared in this section with predictions given by Eurocode 8 (EN 1998-3, 2005). The drift ratio, which is defined as chord rotation capacity at ultimate in Eurocode 8 (EC-8), is given by the following empirical expression:

$$\theta_u = k 0.016 (0.3^\nu) \left[\frac{\max(0.01, \omega')}{\max(0.01, \omega)} f_c \right]^{0.225} \left(\frac{L_V}{h} \right)^{0.35} 25^c (1.25^{100\rho_d}) \quad (1)$$

with	f_c	compressive strength of concrete (MPa)
	ω and ω'	mechanical reinforcement ratio of tension and compression longitudinal reinforcement, respectively
	$\nu = N / bh f_c$	normalized axial force (compression taken as positive)
	b	width of compression zone
	h	cross section side parallel to the loading direction
	$L_V = M / V$	ratio of moment/shear at the end section
	$\rho_{sx} = A_{sw} / bs_h$	transverse steel ratio parallel to the direction x of loading
	f_{yw}	yield stress of stirrups

s_h	spacing of stirrups
A_{sw}	area of transverse steel reinforcement parallel to the direction x within
$k = 0.825$	factor for columns with deformed bars, without detailing for earthquake resistance; $k=0.575$ for smooth bars
ρ_d	geometric ratio of diagonal reinforcement, if any
α	effectiveness coefficient for confinement with stirrups
$c = \alpha \rho_{sx} f_{yw} / f_c$	

If a column is retrofitted with an FRP or TRM jacket in the plastic hinge region, it is logical: (a) to take k equal to 1 instead of 0.825, as the lack of detailing for earthquake resistance has been compensated by the external confinement; and (b) to adopt the expression in Eq. (1) with c given by the sum of two terms: one to account for the contribution of stirrups and a second one to account for the contribution of the jacket, as follows:

$$c = \alpha \rho_{sx} \frac{f_{yw}}{f_c} + \alpha_f \rho_{fx} \frac{f_{fe}}{f_c} \quad (2)$$

with	$\rho_{fx} = 2nt_f / b$	ratio of fibers parallel to the direction x of loading
	n	number of layers of the fiber sheet or textile
	t_f	thickness of one fiber sheet or textile layer
	f_{fe}	effective stress of jacket at conventional failure of the column
	α_f	coefficient for confinement with fibers (TRM or FRP jackets)

$$\alpha_f = \beta \left[1 - \frac{(b - 2R)^2 + (h - 2R)^2}{3bh} \right] \quad (3)$$

where R = radius at corners of the cross section. The coefficient β in Eq. (3) accounts for the reduced or enhanced effectiveness of TRM versus FRP jackets in terms of ultimate strain. On the basis of concentric compression tests on reinforced concrete prisms presented in BOURNAS ET AL. [8], this value is about 0.9. But if jacket failure has not been reached at conventional failure of the column, no reduction (nor enhancement) should be made and β should be taken equal to 1. It should be noted here that in view of the relatively limited experimental database on TRM jacket failures, this value of β should be taken with care. Other materials (e.g. different mortars) may result in different values for the effectiveness of TRM versus FRP. Therefore for such novel materials much more experimental work is needed to propose design values of β .

For columns in which their deformed longitudinal bars have straight ends lapped at the end section of the member, the plastic part of the chord rotation in EC-8 is given by an empirical expression equivalent to Eq. (1), which should be applied with the value of compression reinforcement doubled, as follows:

$$\theta_u^{pl} = k0.0145(0.25^v) \left[\frac{\max(0.01, 2\omega')}{\max(0.01, \omega)} \right]^{0.3} f_c^{0.2} \left(\frac{L_v}{h} \right)^{0.35} 25^c (1.25^{100\rho_d}) \quad (4)$$

If the available lap length ℓ_{ou} is less than a value of $\ell_{ou,min}$, the plastic part of the chord rotation capacity given by Eq. (4) should be multiplied by $\ell_{ou} / \ell_{ou,min}$.

$$\ell_{ou,min} = \frac{d_{bL} f_{yL}}{(1.05 + 14.5\alpha_l \rho_{sx} f_{yw} / f_c) \sqrt{f_c}} \quad (5)$$

with d_{bL} longitudinal bars diameter

f_{yL} yield stress of longitudinal bars

$$\alpha_l = a(n_{restr} / n_{tot})$$

n_{restr} number of lapped longitudinal bars restrained by a stirrup corner or a cross tie

n_{tot} total number of lapped longitudinal bars along the cross section perimeter

For members confined with FRP or TRM, the authors' point of view, also in agreement with BISKINIS [11], is that α_l should be replaced by a term $\alpha_{l,f} = a_f(4/n_{tot})$, contrary to the EC-8 formulation $\alpha_{l,f} = (4/n_{tot})$, as the mechanism of confinement is the same for stirrups and composite jackets and the effectiveness coefficient must be also considered in FRP or TRM jacketed members.

The value of the chord rotation at yielding θ_y added to the plastic part to obtain the total chord rotation capacity can be estimated from the following expression proposed in EC-8:

$$\theta_y = \phi_y \frac{L_v}{3} + 0.0013 \left(1 + 1.5 \frac{h}{L_v} \right) + 0.13 \phi_y \frac{d_{bL} f_{yL}}{\sqrt{f_c}} \quad (6)$$

where the yield curvature ϕ_y can be predicted empirically according to BISKINIS [11], as $\phi_y = 1.55 f_{yL} / E_s d$. Equation (6) initially derived for the calculation of θ_y for columns with continuous bars, is also applicable to columns with lap splices, with the yield moment and the

yield curvature ϕ_y computed with a compression reinforcement doubled over the value applying outside the lap splice. If the straight lap length ℓ_o is less than $\ell_{oy,min} = 0.3d_{bL}f_{yL}/\sqrt{f_c}$, then: M_y and θ_y should be calculated with yield stress f_y multiplied by $\ell_o/\ell_{oy,min}$. The second term of Eq. (6) should be multiplied by the ratio of the value of yield moment M_y as modified to account for the lap splicing, to the yield moment outside the lap splice.

For the geometric and material properties of the columns with either continuous bars or with lap splices tested in this study, the predicted and experimentally measured drift ratios at failure are presented and compared in Table 4 for all retrofitted and unretrofitted specimens. The predicted drift ratios at failure according to the EC8 - based approach described above are 31% and 42-43% lower than the experimental values for the unretrofitted and retrofitted columns with smooth bars, respectively. Hence the EC-8 - based formulation presented above is conservative, for columns with smooth bars, especially for members jacketed with FRP or TRM. This is possibly attributed to the low value of factor k ($k=0.575$). For columns with continuous deformed bars the predicted drift ratios at failure according to the EC-8 - based approach are 14% higher than the experimental value for the unretrofitted specimen (L0_C) and 20% lower than the experimental value for the specimen retrofitted with glass fiber TRM jacket (L0_M4G). For specimen L0_M4 the predicted drift ratio is practically equal to the experimental one, but the latter was determined at the end of the test (the reduction of peak resistance was equal to 88% of the maximum recorded resistance) and not at conventionally defined failure. No comparison can be made for specimen L0_R2 as its deformation capacity was controlled by the unconfined length above the FRP jacket. It can be concluded that the EC8 - based formulation, as modified here, is in moderate to good agreement for members with continuous deformed bars jacketed with TRM or FRP.

Finally, for columns with deformed lap spliced bars the EC-8 predicted drift ratios (with the modified value for a_{lf}) are in quite good agreement for FRP and TRM jacketed members with shorter lap lengths. This is not the case in columns with longer lap splices, where the EC-8 - based formulation presented above was found to be quite conservative, as the 6.02% predicted drift is far from the ultimate drift at failure. Note here that when the test was terminated at a drift ratio of 7.81% the average reduction of peak resistance for specimens L40d_R2 and L40d_M4 was only 94% of the maximum resistance.

Table 3: Comparison between EC-8 -based Predictions and Experimentally Measured Drift Ratios at Failure.

Specimen notation	Experimental drift ratio at "failure" (%)	Predicted drift ratio at "failure" (%)	Predicted/ Experimental
L0S_C	3.75	2.60	0.70
L0S_R2	7.81	4.30	1.81
L0S_M4	7.81	4.46	1.75

L0_C	3.43	3.91	1.14
L0_R2	5.15*	7.51	1.45
L0_M4	7.81	7.79	0.99
L0_M4G	7.20	5.73	0.80
L20d_C	3.59	2.62	0.73
L20d_R2	5.78	5.51	0.95
L20d_M4	5.00	5.59	1.12
L40d_C	3.28	4.66	1.42
L40d_R2	7.81 (stroke end)	6.02	No failure
L40d_M4	7.81 (stroke end)	6.02	No failure

* Controlled by failure at the unconfined length, outside the jacket

5 Conclusions

The effectiveness of TRM jackets as a means of confining RC columns with limited capacity due to buckling of the longitudinal bars or due to bond failure at lap splice regions is investigated in this study. Comparisons with equal stiffness and strength FRP jackets allows for the evaluation of the effectiveness of TRM versus FRP. The ten tests on full scale columns under cyclic uniaxial flexure show that TRM jackets are quite effective as a means of increasing the cyclic deformation capacity and the energy dissipation of old-type RC columns with poor detailing, by delaying bar buckling or by preventing splitting bond failures at columns with inadequate lap splices. More specific conclusions are summarized in a rather qualitative manner as follows:

- The seven tests on columns with continuous longitudinal reinforcement (Series L0S_... and L0_...) show that TRM jackets are quite effective as a means of increasing the cyclic deformation capacity and the energy dissipation of old-type RC columns with poor detailing, by delaying bar buckling. Compared with equal stiffness and strength FRP, TRM jacketing has practically the same effectiveness for columns with smooth rebars, but higher effectiveness by about 50% for columns with deformed rebars.
- From the six tests on columns with lap-spliced longitudinal reinforcement (Series L20d_... and L40d_...), it may be concluded that TRM confining jackets provide substantial gain in lateral strength and deformation capacity of cyclically loaded reinforced concrete columns with lap splices at the columns' base. Compared with equal stiffness and strength FRP jackets, they are characterized by a slightly reduced effectiveness in terms of deformation capacity for columns with short lap splices and with the same effectiveness for columns with longer lap lengths.
- The EC-8 - based modified formulation is in moderate to good agreement for members with continuous deformed bars jacketed with TRM or FRP, and conservative for columns with smooth bars due to the low value of the factor k . For columns with deformed lap spliced bars the EC-8 predicted drift ratios are in good agreement for FRP and TRM

jacketed members with shorter lap lengths, while its predictions are quite conservative in the case of columns with longer lap splices.

Despite their relatively limited number, all test results presented in this study indicate that TRM jacketing is an extremely promising solution with great potential for the confinement of poorly detailed reinforced concrete columns in seismic regions. Hence future research should be directed towards providing a better understanding of parameters including the level of axial load, initial column damage, different shear spans, different loading histories, other cross sections, and the effectiveness of TRM versus FRP for seismic retrofitting after fire exposure.

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